STRUCTURAL RESPONSE TO LONG-DURATION EARTHQUAKES

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16. ABSTRACT

The effects of postulated Cascadia subduction zone earthquakes on inelastic structural response have been quantified. The earthquakes studied ranged in size from those previously recorded to the largest plausible event, a magnitude 9.5, 240 second duration earthquake. Artificial acceleration records attenuated to epicentral distances corresponding to coastal range sites and Puget Sound sites were generated. These records were used as input for inelastic response history analyses of single-degree-of-freedom systems with either bilinear or degrading stiffness hysteretic relationships. The results indicate that the maximum displacements are not significantly greater than those produced by previously recorded events or by records that are compatible with current design code response spectra. However, the inelastic energy dissipated and the numbers of displacement cycles are somewhat greater for the largest events, although the energy demands and cyclic demands are similar to those from previous events for magnitudes up to 8.5. Since, the maximum credible event is not well established at this time no changes to the current design procedures are recommended.

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CHAPTER 1

SUMMARY

Recently discovered evidence suggests that the Cascadia subduction zone (CSZ) located off the Pacific Northwest coast is capable of generating large-magnitude, long-duration earthquakes. The seismic design procedures, currently in use in the region do not recognize these events. The purpose of this research was to estimate potential structural demands from such events and to compare them with what is currently used in design.

Since no acceleration records are available for CSZ earthquakes, artificial records that corresponded to magnitude 7.9 to 9.5 events were generated. These were used as input for inelastic, response history analyses for single-degree-of-freedom structures with load-displacement relationships indicative of steel and reinforced concrete. The yield levels were varied to determine the effect of lateral strength, and the period was varied to account for different structure stiffnesses. The response was quantified with maximum displacement, hysteretic (inelastic) energy dissipated and the numbers of cycles experienced at various displacement levels.

The CSZ earthquakes with magnitudes smaller than 8.5 -a probable upper bound- that were attenuated for Puget Sound sites produced demands that were comparable to previously recorded events, such as the 1940 El Centro and 1949 Olympia. Both of these recorded events have been important in the establishment of the current design procedures.

Response for long-period structures was elastic. Since the greatest uncertainty regarding potential ground motion exists for the long-period range, this result is positive.

For the largest magnitude events, the displacements are probably tolerable, since they are only slightly larger than those from the previously recorded events.

However, the energy demands and the cycle counts are substantially larger. Many of these demands occur as low-amplitude cycles, which correspond to a range of loading not typically investigated in the laboratory.

The amplification produced by soil was investigated for several cohesionless soil types. The amplification in inelastic response is not always consistent with that for elastic response spectra. Thus, no general rule could be formulated regarding the effects of soils on inelastic demands.

CHAPTER 2

CONCLUSIONS AND RECOMMENDATIONS

2.1 Conclusions

In this research, an evaluation of the response-history results showed that damage for the largest magnitude, long-duration CSZ earthquakes may be higher than that predicted by current design code criteria and higher than the damage demands produced by past Western U.S. strong ground motions. However, for the lower range events, those thought to be more probable, the demands may be similar to those predicted using current code criteria and existing ground records.

CSZ events may produce demands that have greater numbers of cycles in the low-amplitude range. This is attributable to the attenuation of motion to the Puget Sound region and to the longer durations. Structures at sites along the coast would experience more damage. Long-period structures tend to have lower inelastic demands than stiffer structures, and the response of such structures in many cases is elastic. Soils tend to amplify the structural response, although no general rule for predicting such amplification was found.

2.2 Recommendations

As a result of this research the following are recommended:

1. Because the results are sensitive to the assumed events, as much as possible should be determined about the postulated events. The current results are based upon the best current estimates of shaking possible from the CSZ. However there exists a great deal of uncertainty regarding the largest credible and the probable earthquakes of this region.

2. Until more data is amassed regarding the largest possible events, the current design code force and detailing provisions should be retained. Some

researchers believe that a magnitude 8.5 event is the maximum credible event. This work showed that the demands from such an event are not dramatically different from the levels currently expected.

3. The information from this work can be used to estimate potential CSZ demands on structures when the need arises. This may be accomplished for either existing or new structures following the guidelines provided.

4. The following additional research should be pursued: a) Determine typical as-built relationships between structure yield force and weight; b) Using the range of values found in a) conduct additional analyses with a finer grid of structure strengths; c) Include degrading strength as a parameter in such analyses; d) Investigate, in the laboratory, the ability of typical substructure details to endure large numbers of small-amplitude displacement cycles and e) Determine how the existing work may be used for multiple degree-of-freedom structures.

CHAPTER 3

INTRODUCTION

3.1 Problem Statement and Scope

The Pacific Northwest is located near the Cascadia subduction zone (CSZ), a tectonic feature thought to be capable of producing long-duration, high-intensity earthquakes. The seismic risk in such areas to engineered structures, including bridges, may be closely tied to the duration of strong shaking. Current geophysical and geological research is establishing the possibility and probability of the occurrence of such large, long-duration events. In modern times such events have not occurred in the Northwest region, and as a result, the current seismic design criteria do not reflect the possibility of long-duration events. For the most part, seismic design criteria have evolved from the experience of California and, in part, from the experience of Western Washington. These criteria include both the explicit code provisions and the inferred performance levels taken from experimental studies of structures and sub-assemblies. While the seismic performance levels expected from designs based upon the current provisions may be adequate for earthquakes of the type that have been experienced in modern times, there is no assurance that the performance of these same designs in long-duration events would be satisfactory.

This research investigated the effects that such postulated earthquakes could have on the inelastic response of structural systems. The research focused on the inelastic response of single-degree-of-freedom (SDOF) oscillators subject to both artificial and previously recorded strong-motion acceleration histories. This analytical modeling included: various yield force-to-weight ratios, bilinear (elastic-plastic) response, stiffness degradation and various site soil conditions. The objective was to contrast the expected seismic response of structures subject to the

more common, and therefore better understood, short-duration events with that expected from the postulated longer-duration events. Based on the comparisons recommendations are made to the WSDOT for the design of structures subject to long-duration shaking.

CHAPTER 4

BACKGROUND

4.1 Seismicity of Western Washington

The tectonic phenomenon thought to be capable of producing large subduction earthquakes in the Pacific Northwest is the subduction of the Juan de Fuca plate beneath the North American plate in the Cascadia subduction zone. See Figure 1. According to McCrumb, et al (1) the Juan de Fuca plate is moving in a northeasterly direction at a rate of 3.5 to 4.5 cm per year relative to the North American plate. The intersection between the two plates is approximately 1000 kilometers long. The tectonic features of the CSZ are similar to other subduction zones around the world that have experienced large earthquakes (2,3).

Some researchers postulate that the plates are locked and accumulating strain energy (4,5). Other research has pointed to an unlocked interface and aseismic action (6,7) The area of locked interface is important, since larger locked interface areas are thought to lead to larger rupture areas, which in turn infer larger earthquakes. Until more conclusive evidence concerning the locking of the plate interface is produced, there is much uncertainty in the postulated subduction earthquake ground motion.

Recorded seismic activity in the Pacific Northwest has been attributed to the action of the subducting of the Juan de Fuca plate. Noson, et al (8) and McCrumb, et al (1) report that there have been over 20 damaging earthquakes in the past 150 years. Some earthquakes are believed to have been caused from the fracturing of the Juan de Fuca plate, while other earthquakes have been caused by the shifting and settling of the North American plate as it reacts to the subduction process. There have been no large CSZ earthquakes recorded in modern times.

Geologic evidence of large CSZ earthquakes has recently been found by Atwater (9) and others listed by Heaton and Hartzell (3). This evidence includes sand boils indicating liquefaction of buried sands, subsidence similar to that found in Alaska after the 1964 earthquake, sand deposits similar to tsunami deposits found in Chile, sedimentary deposits on the continental slope, and apparent concurrence of several prehistoric landslides. These findings point to past large CSZ earthquakes with recurrence intervals ranging from several hundred to several thousand years.

4.2 Hypothetical Ground Motion in the Pacific Northwest

Since no acceleration histories of large CSZ events have been recorded, it was necessary to generate artificial records for this research.

Acceleration information for subduction events exists, but only for events smaller than the largest plausible CSZ event. Very large events have been experienced worldwide, for instance the 1960 Chilean event (moment magnitude, M_w = 9.5) and the 1964 Alaskan event, M_w = 9.2 (10). Unfortunately no acceleration information was obtained from either of these events.

Heaton and Hartzell (11) have used kinematic modeling to build accelerograms for large CSZ earthquakes. They postulated that the maximum credible earthquake that could occur would be similar to the 1960 Chilean event, and could have a total duration of approximately 240 seconds. In addition, they used the elastic response spectra for 56 recordings of shallow subduction events with magnitudes between 7.0 and 8.5 to provide a lower bound estimate of potential shaking. Then linear interpolation between the maximum event (Mw = 9.5) and the smaller events was used to estimate the elastic spectra for magnitude 8.5 and 9.0 events.

Crouse (12) developed median response spectra for CSZ events using regression techniques on a data base containing 237 response spectra from subduction earthquakes worldwide. The largest event had a magnitude of 8.2. His regression equation included both epicentral distance and focal depth so that spectra could be estimated at any location. Crouse's spectra are shown along with those from Heaton and Hartzell's work in Figure 2. Both sets of spectra were developed for coastal range sites.

Most data for earthquake duration is for the more common smaller events. Chang and Krinitzky (13) provide upper bound estimates of duration for both rock and soil sites (Table 1). They used bracketed duration, which is the time between the acceleration's first and last exceedence of 5 percent of gravity. Donovan (14) also provided a linear regression for duration as shown in Figure 3. Comparison of the information in Table 1 and this figure shows that Donovan's equation is bounded by the estimates for rock and soil. Note also that the two predictions agree less well in the large magnitude range.

There is little definitive information describing the duration of these hypothetical large subduction zone earthquakes. Heaton and Hartzell cite 240 seconds total duration as an upper bound. Accounts of the shaking in the 1964 Alaskan event led Housner and Jennings (15) to conclude that the duration of strong shaking lasted 2 minutes and the total duration was 4 minutes. Similarly, witnesses reported to St. Amand (16) that the largest of the 1960 Chilean shocks lasted 3.5 minutes.

Table 1 Durations for Different Earthquake Magnitudes (after (13))

	Duration (sec)*	
Magnitude	Rock	Soil
5.0	4	8
5.5	6	12
6.0	8	16
6.5	11	23
7.0	16	32
7.5	22	45
8.0	31	62
8.5	43	86

* using bracketed duration

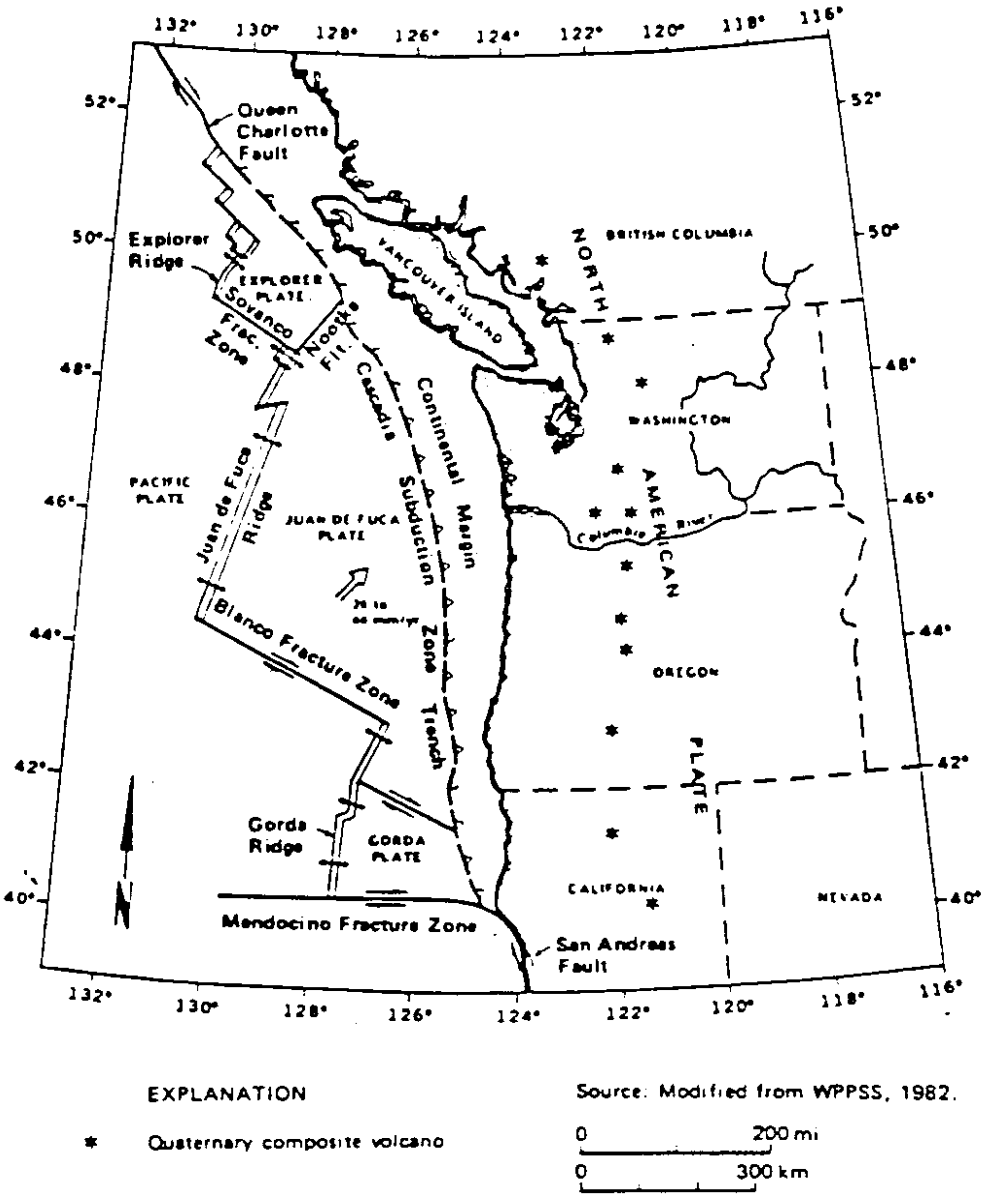


Figure 1 Plate Tectonic Structure in the Pacific Northwest (from (1))

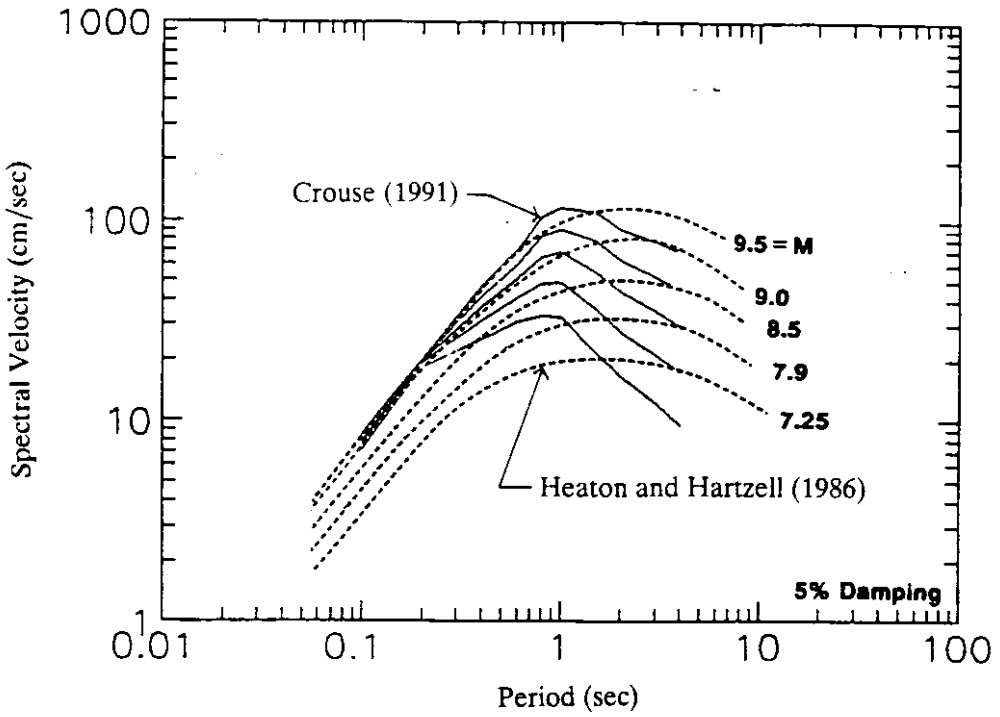


Figure 2 Average and Median CSZ Spectra for Coast Range Sites; Heaton & Hartzell (dashed), Crouse (solid); (from (12))

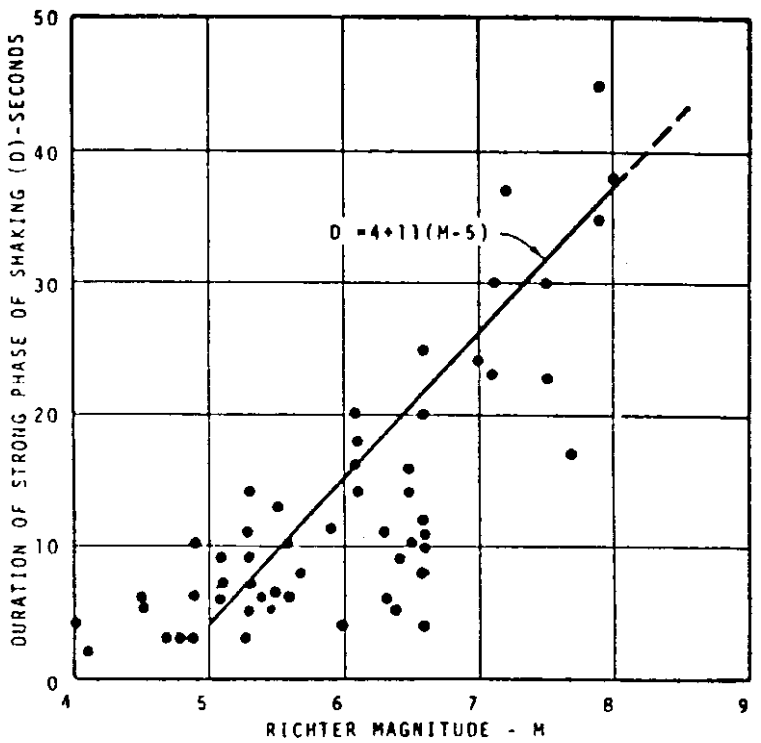


Figure 3 Relationship Between Magnitude and Duration of Strong Phase of Shaking (after (14))

CHAPTER 5

PROCEDURES

5.1 Artificial Acceleration Record Generation

The artificial acceleration histories used to represent potential CSZ shaking were generated using SIMQKE (17), which produces acceleration records that are compatible with user specified target elastic response spectra. These "compatible" records then have elastic response spectra that are within specified tolerances of the input spectra. SIMQKE also requires an input envelope function to control the shape and duration of the resulting acceleration history.

The target response spectra used in this research were those of: Crouse (12) and Heaton Hartzell (3). Since the largest inventory of structures in Washington State is located in the Puget Sound region, an epicentral distance of 75 kilometers and a focal depth of 32.5 kilometers were used in Crouse's regression equation to generate Puget Sound spectra. Plots of the response spectra used in this study are shown in Figure 4. Heaton and Hartzell's spectra were used for generation of magnitude 9.5 giant earthquakes. These spectra and the resulting acceleration records are considered to be upper bounds possible from CSZ earthquakes.

To generate earthquakes that represent ground motion of Washington soil sites, Crouse's spectra for median earthquakes were amplified by Tsiatas, et al's (18) spectral acceleration amplification factors. Soil Groups 1 and 5, as defined by Tsiatas, et al. were used in this study. Group 1 is a medium to dense cohesionless soil less 20 to 50 feet thick. Tsiatas showed that this soil type has the largest spectral amplification factors, for periods below 0.4 seconds. Group 5 soil is a medium to dense cohesionless soils 50 to 100 feet thick, and they have the largest amplification factors in the period range of 0.6 to 1.0 seconds.

A compound acceleration envelope was used in SIMQKE to control the overall form of the artificial accelerograms. The envelope consists of a parabolic rise section, a level section and an exponential decay section, which was set to produce a final amplitude of 2 percent of the level portion. Time lengths of the rise, level and decay portions of the envelope were proportional to those of Heaton and Hartzell's maximum acceleration history, for which the total duration was 240 seconds, the rise portion was 42 seconds, the level time 48 seconds and the decay was 150 seconds. For other total durations, the lengths of the rise, level and decay were proportional to that of the 240 second record. An example for a 60 second overall duration is shown in Figure 5.

A matrix of magnitudes and overall durations was created to reflect realistic earthquakes. See Figure 6. The matrix also includes enough records to determine the sensitivity of response to duration. The effective durations of CSZ artificial earthquakes, generated by SIMQKE, were compared with Chang and Krinitzsky's work. The magnitudes used by Chang and Krinitzsky were Richter magnitudes and not moment magnitudes as used by Crouse and Heaton Hartzell. Because Chang and Krinitzsky's data was below magnitude 8 where moment magnitude and Richter magnitude are similar, the difference in magnitude scales was considered to be small.

It should be noted that all of the magnitude and duration combinations were created for Puget Sound sites. However, not all of the combinations were created for coast range sites. Only magnitude 7.9 and 8.5 earthquakes were created for coast range locations. Results from these coast range earthquakes were compared to Puget Sound earthquakes of the same magnitude and duration to determine response sensitivity to attenuation of ground motion.

5.2 Response History Analyses and Structural Models

Two structural models were used: a bilinear model to emulate steel behavior, and a degrading stiffness model to emulate reinforced concrete behavior. Only single-degree-of-freedom (SDOF) models with 5 percent of critical damping were used. Strain hardening was set at 5 percent of the original stiffness.

Yield-force-to-weight ratios (Fy/W) of 0.125, 0.25, and 0.50 were used to determine the sensitivity of the response to structural strength. The yield-force-to-weight ratio is defined as the quotient of the lateral force that causes yielding in the structure and the structure weight. Typically, a plastic collapse or "push-over" analysis is required to determine the load-displacement curve.

The response of the bilinear model was determined using the program SPECTRUM (19), which performs a response-history analysis of a structure subject to a specified acceleration record. The load-displacement relation used in this model is shown in Figure 7.

For the degrading stiffness model, the program DRAIN2D (20) was used. The DRAIN2D model allows no strength degradation, no pinching, but degrades in stiffness based on the maximum displacement attained. See Figure 8.

5.3 Quantities used for Comparisons and Evaluations of Results

The output of the response history analyses was used to generate inelastic maximum displacement spectra for a range structure periods. However, such maxima provide only point-in-time estimates of the demands on the structure. To quantify cumulative demands on the structure, the inelastic energy dissipated during the earthquake was also calculated. This energy is calculated by determining the area under the spring force-displacement curve. To exclude the elastic strain and kinetic energy stored, comparisons were made only after the structure had returned to a rest position.

In addition to the inelastic energy dissipated, the distribution of inelastic cycle amplitudes throughout the response history also provides insight into the structural demands. The numbers of inelastic half-cycles, quantified in terms of displacement ductility demand, were counted for the analyses using the rain flow method (Dowling, 21). This method accounts for the effects of varying cyclic amplitude, frequency, and random sequencing of the cycle peaks. Since laboratory testing is based upon application of specific numbers of cycles at selected ductility levels, the cycle counts determined from the response analyses can be used for comparison with typical testing sequences.

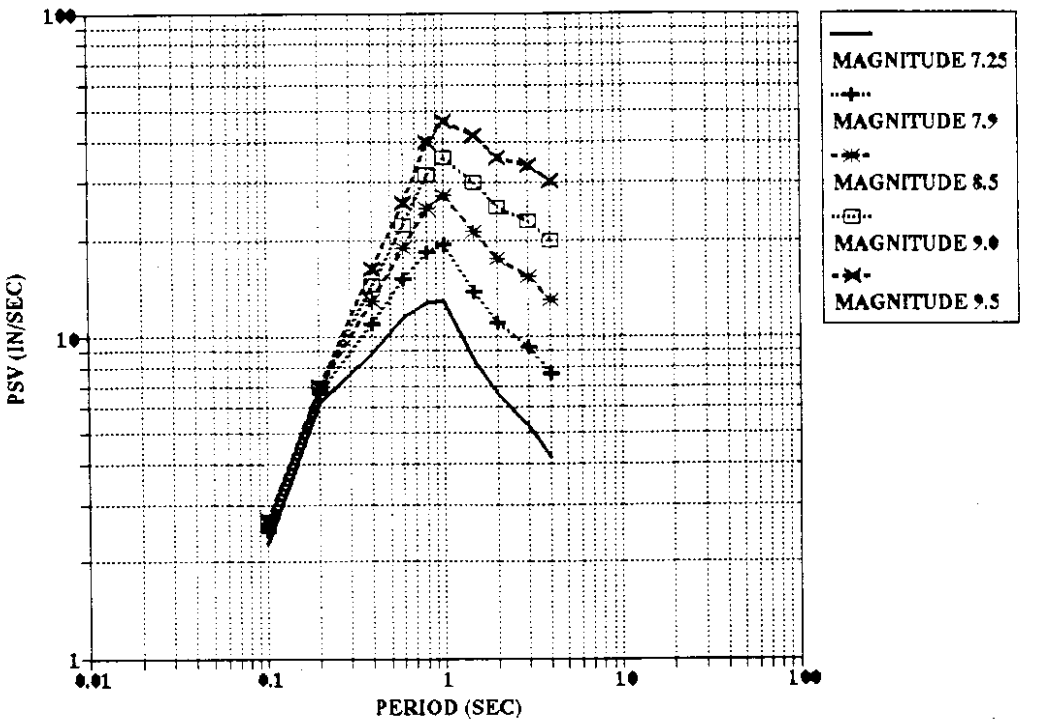


Figure 4 Median PSV Spectra Estimated for Puget Sound Sites (after (12))

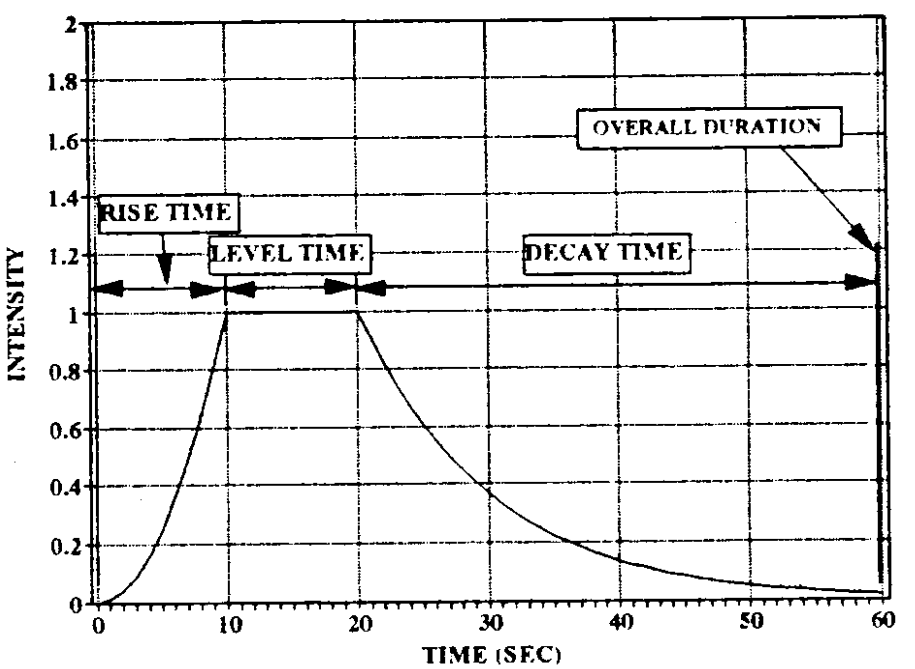


Figure 5 Acceleration Envelope Used in SIMQKE

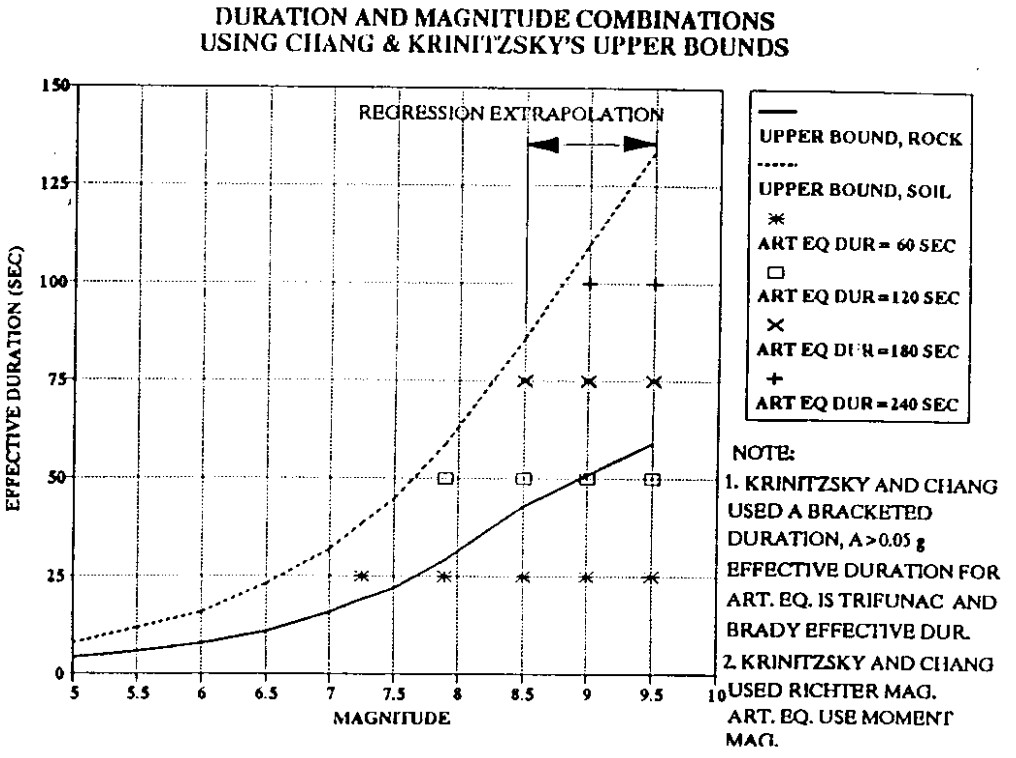
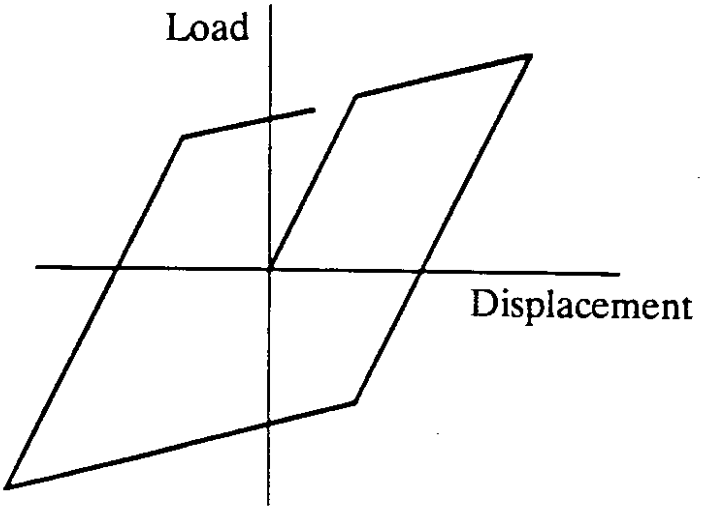


Figure 6 Magnitude - Duration Matrix



Bi-linear Elasto-Plastic
(Steel)

Figure 7 Bilinear Model Force-Displacement Relationship

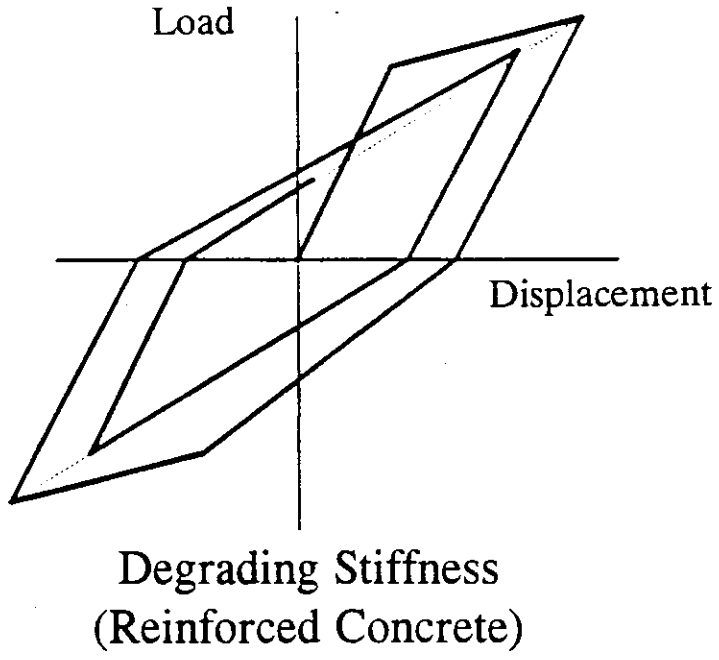


Figure 8 Degrading Stiffness Model Force-Displacement Relationship

CHAPTER 6

DISCUSSION OF RESULTS

The primary structural type in the WSDOT bridge inventory is reinforced concrete. Therefore, the results discussed in this chapter are primarily those for the degrading stiffness structural model.

6.1 Maximum Displacements

The maximum total displacements for structures in the Puget Sound basin with yield force to weight ratios (Fy/W) of 0.25 and 0.125 are shown in Figures 9 and 10. Results for both artificial records and actual records are shown as functions of elastic vibration period. Also shown for reference are lines indicating displacements corresponding to ductility demands of one and four. Ductility is defined as the maximum displacement divided by the displacement at yield.

It is seen that the demands for the Fy/W=0.25 structures do not exceed a ductility demand of four even for the magnitude 9.5 event, and in fact, the highest demands are caused by the 1940 El Centro record. For Fy/W=0.125 the ductility demands for all ground motions are less than four for structures with periods greater than 0.8 seconds. For structures with periods greater than 0.3 seconds the ductility demands are less than four for the magnitude 8.5 and smaller events. It should be noted that structures with low period and low Fy/W ratios are not common since the design spectra have higher values in the low period range. Thus the demands shown in Figure 10 that exceed a ductility demand of four are not necessarily alarming.

The figures also indicate that the response for long period structures - those with periods of vibration longer than about 2 seconds - is either elastic or nearly

elastic. This is important because the ground motion estimates are the most uncertain in the long period range.

Figure 11 shows the total displacements for Fy/W=0.25 structures subject to three different magnitude 8.5 artificial acceleration histories. It is seen that the maximum response is nearly identical for all three records. This occurs because the artificial records are forced to match a target response spectrum. These results shown are for the bilinear structural model rather than the degrading stiffness model; the results of which are similar.

Shown for the bilinear model in Figure 12 are comparisons between the displacement demands for structures located along the coast and those in the Puget Sound basin when subject to the magnitude 8.5 record. The difference in excitation levels for the two locations is the result of attenuation of ground motion. The total displacements are not significantly different for the two locations except for the lowest Fy/W ratio in the low period range.

6.2 Inelastic Energy Demands

The inelastic energy demands for Fy/W ratios of 0.25 and 0.125 are shown in Figures 13 and 14 for the degrading stiffness models and the same ground motions as those in Figures 9 and 10 above. The energy demands have been normalized by the strain energy stored at first yield of the structure. This normalization provides a consistent measure of energy demand in the same manner that ductility does for displacement demand.

It is seen from the plots that much more energy is required of structures with low Fy/W ratios as evidenced by the different ordinate maxima. As expected, the large magnitude events produce higher energy demands than do the smaller events. When compared with the demands from the two previously recorded events, El Centro and Olympia, two trends are evident: 1. only the two largest events cause

significantly more demand than the El Centro record and 2. essentially all the artificial events cause more demand than the Olympia record.

The elastic response of long-period structures is apparent for structures with periods greater than two seconds since the inelastic energy demand is zero.

Some researchers suggest that the magnitude 8.5 event is a more probable upper bound event for the CSZ, since a magnitude 9.0 or 9.5 event would require the entire plate interface to rupture. From consideration of the both the maximum displacement and energy demand plots the following inference may be drawn: If the a structure were designed and detailed to withstand the El Centro record without collapse then it would probably endure the magnitude 8.5 CSZ event without collapse. This is important because the El Centro record has been instrumental in the development of design and detailing practices aimed at preventing collapse.

6.3 Half-Cycle Counts

Figures 15 and 16 show half-cycle counts for Fy/W ratios of 0.25 and 0.125 for degrading-stiffness structures with a period of 0.6 seconds. This period was selected as one typical of those seen in bridge construction. The counts are plotted as functions of displacement ductility ranges. For example, a count of 20 in a range of 2.5 to 3 indicates that over the entire response history 20 displacement excursions occurred that were larger than 2.5 times the yield displacement, but less than 3 times the yield value.

It is seen in the figures that the weaker structures (Fy/W=0.125) experience more half-cycles at nearly all demand levels than do the stronger structures. In fact for the stronger structures there are no demands higher than a ductility of 5.5, whereas there are as many as five cycles at similar demand levels for the weaker structures subject to the two largest events. It is also apparent from the figures that numerous cycles occur in lower demand ranges.

To provide a perspective of the half-cycle counts for several periods the counts were plotted for the magnitude 8.5 event as shown in Figures 17 and 18. In these figures typical half-cycle counts from laboratory testing are also shown. The laboratory counts are based on the application of three cycles at evenly spaced ductility demand levels. For Fy/W of 0.25 structures the laboratory test sequence exceeds the demands for the earthquake for all five periods and all the demand ranges above 3.5. On the other hand, the laboratory sequence is less severe for all the weaker structures with periods under about 1.0 seconds.

As a point of clarification, it should be recognized that a half-cycle, for instance, of magnitude eight is equivalent to laboratory cycling at ductilities of four, since laboratory test levels are quantified by the maximum displacement in one direction. Thus half-cycles applied to a demand of four are typically followed by reversal of loading to a demand of negative four. The difference between the two extreme displacements corresponds to a demand of eight.

6.4 Bilinear vs. Degrading Stiffness

The inelastic displacement spectra for the bilinear and degrading stiffness models are compared in Figure 19. Over much of the period range, there are only slight differences in the inelastic displacement response of the these two models, although there is as much as 20 percent difference in the response for structures with periods less than 0.5 seconds. The trend, however, is not constant over all the periods in this range. Above periods of about 2 seconds, the response is elastic for both models and is thus identical for a given ground motion.

The differences in hysteretic energy demand for the two models are shown in Figure 20. As seen in the figure the degrading stiffness structures universally dissipate more energy than do the bilinear structures. The differences are as large

as 300 percent for the two models. The probable cause of the increase is the fact that the degrading stiffness models, after experiencing first yield, continue to dissipate energy even if subsequent displacements are smaller than the yield value (See Figure 8). The implication of these results is that the displacements tend to be relatively insensitive to the model used, while the energy demands are sensitive to model type.

6.5 Soil Effects

Ground motions based on spectra for two Washington State soil groups were used to quantify the amplification effects of these soils on the inelastic response of the bilinear models. One soil type (WSDOT Group 1) was a shallow, firm, cohesionless material and the other (WSDOT Group 5) was a deep, firm conhesionless material. Only the results for the Group 5 soil are discussed in this report. For the Group 1 results, the reader is referred to the Technical Report.

Figure 21 gives the inelastic displacement spectra for the magnitude 7.9 and 8.5 events after attenuation to the Puget Sound region. The results for a Fy/W ratio of 0.125 are shown for both the Group 5 soil and the "standard" firm soil. For the smaller event, the amplification of displacement is most apparent for structures with periods below about 0.5 seconds. In this range the trends are not consistent; however, the maximum amplification of the inelastic response is about 100 percent. For the larger, magnitude 8.5 event, the difference in response extends over most of the periods, but the maximum amplification remains about 100 percent. For the smaller event, ductility demands greater than 4 do not occur above a period of 0.4 seconds. However for the larger event, ductility demands over 4 occur up to periods of 0.7 seconds.

The hysteretic energy demands for the Group 5 soil type are shown in Figure 22 for the same combination of parameters. For the smaller event the

increases in demand are modest over all the periods. However for the larger event, increases as large as 200 percent occur between periods of 0.3 and 1.0 second.

It is apparent that the increases in the displacement do not correspond to the increases in energy demand, and the trends in either quantity vary over the periods considered. The same observation is apparent for the Group 1 soils, as well. Thus the presence of deep soil layers may amplify both displacement and hysteretic energy demand, as is well known, although no general rule can be formulated based on these tests.

6.6 Input Spectra Effects

Artificial acceleration records based on Heaton and Hartzell's upper bound spectra for a magnitude 9.5 earthquake, are more intense that the records based upon Crouse's spectra for median earthquakes. Likewise structural inelastic response is expected to be correspondingly more intense. Figure 23 shows a comparison of inelastic displacement for records based on Heaton and Hartzell's spectra and those based on Crouse's median spectra. Figure 24 shows the corresponding energy demands. Both figures are for bilinear structures with Fy/W ratios of 0.25.

For Puget Sound sites and structures with Fy/W of 0.25 and periods less than 0.5 seconds, the displacement demands are approximately twice as large for the upper bound spectra as for the median spectra. Above 0.5 seconds, the differences are not as conclusive. For the same set of parameters, the hysteretic energy demands for the upper bound event are as great a 20 times those for the median event. The large differences in the demands reflect the differences in the spectra on which the records are based.

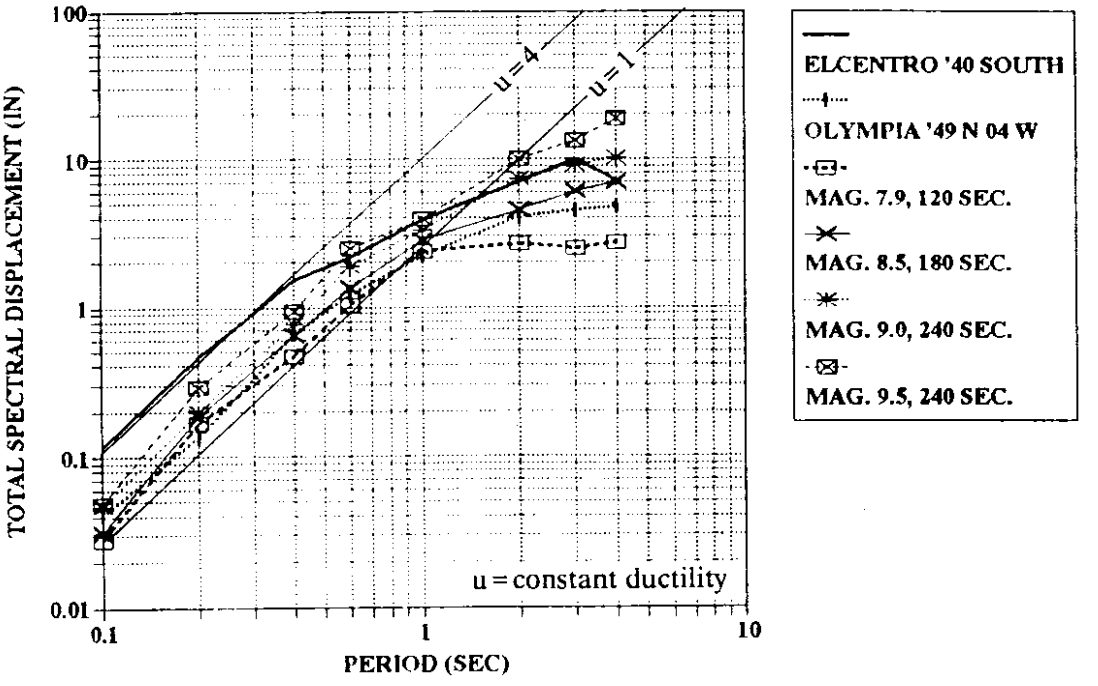


Figure 9 Maximum Displacements - Puget Sound, Fy/W = 0.25, Degrading Stiffness

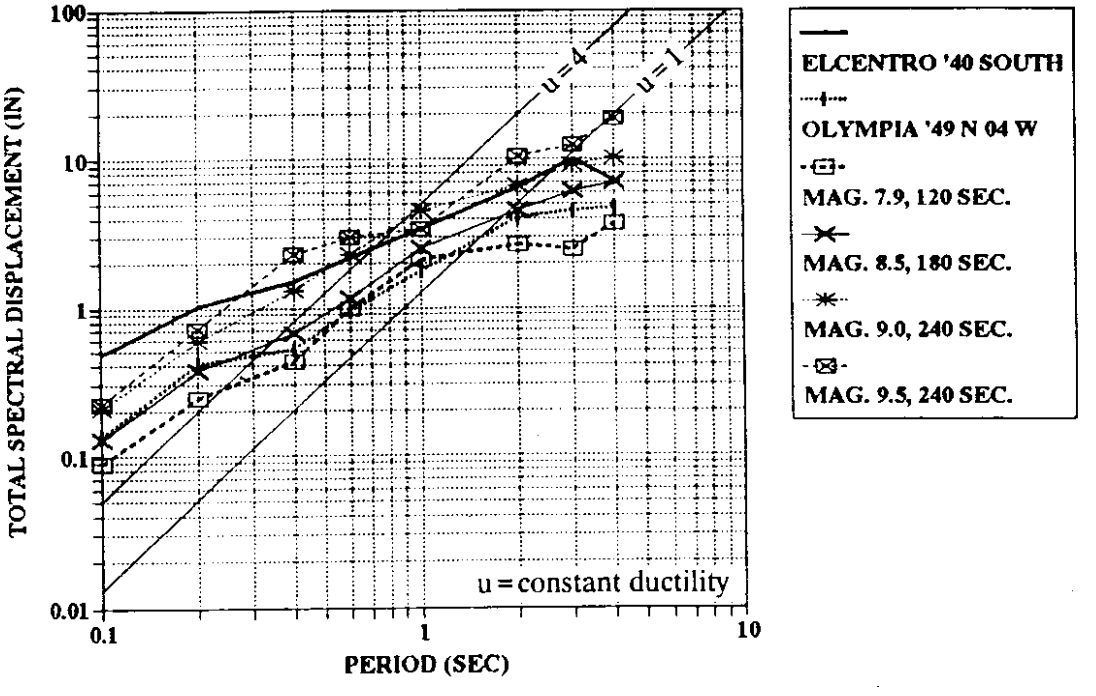


Figure 10 Maximum Displacements - Puget Sound, Fy/W = 0.125, Degrading Stiffness

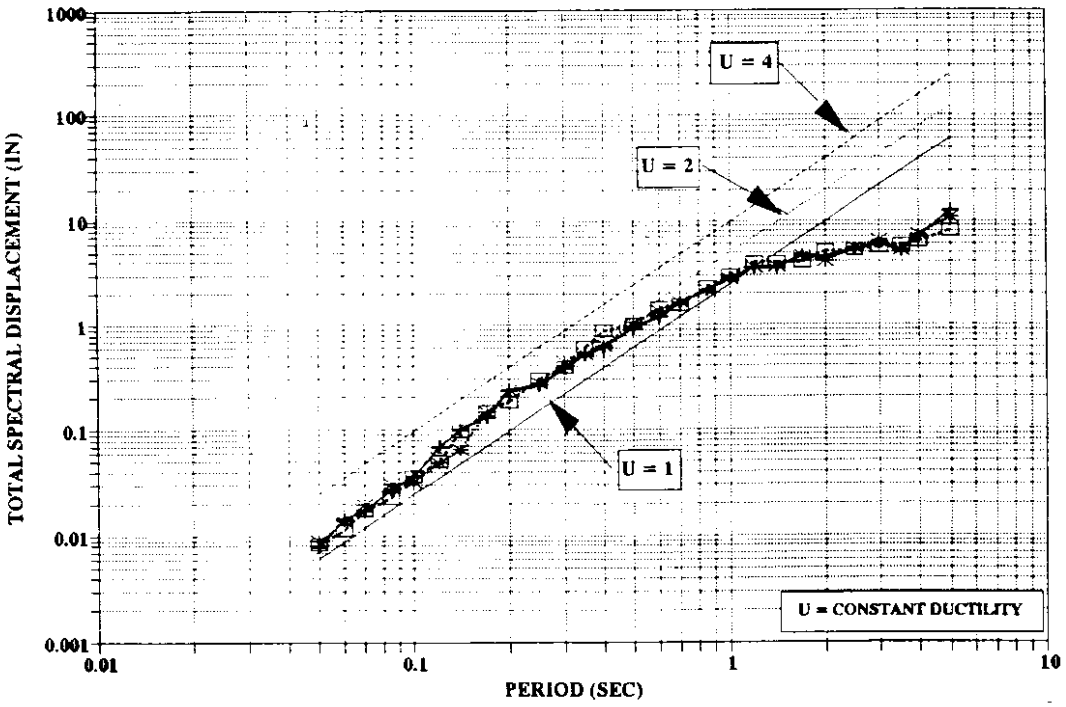


Figure 11 Maximum Displacements for Three Different Records - Puget Sound, Magnitude 8.5, Fy/W = 0.25

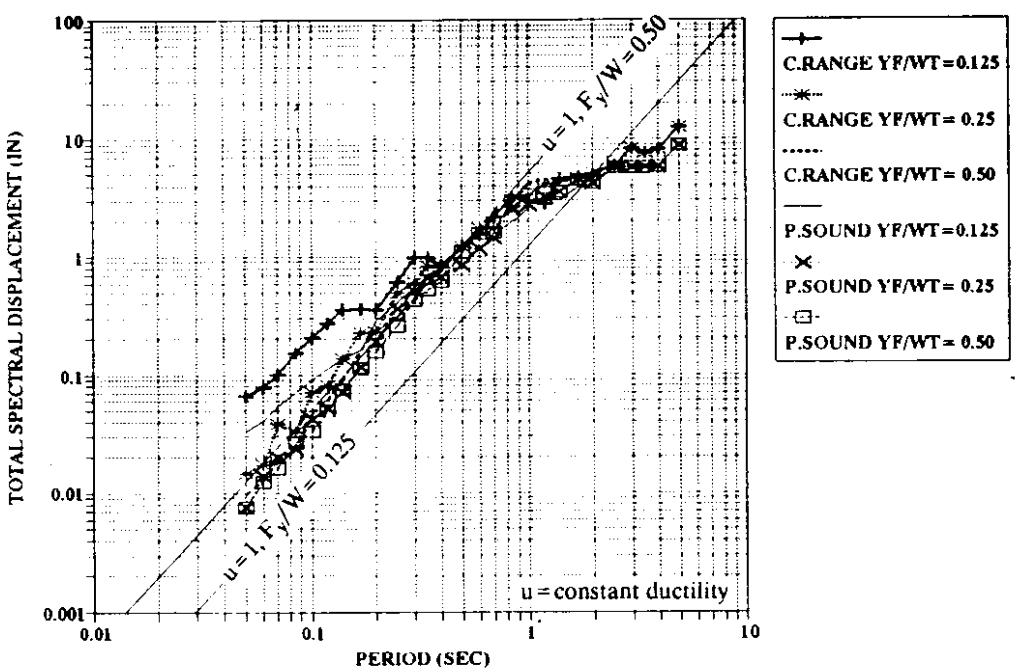


Figure 12 Maximum Displacements - Coast Range vs. Puget Sound, Magnitude 8.5, Bilinear Model

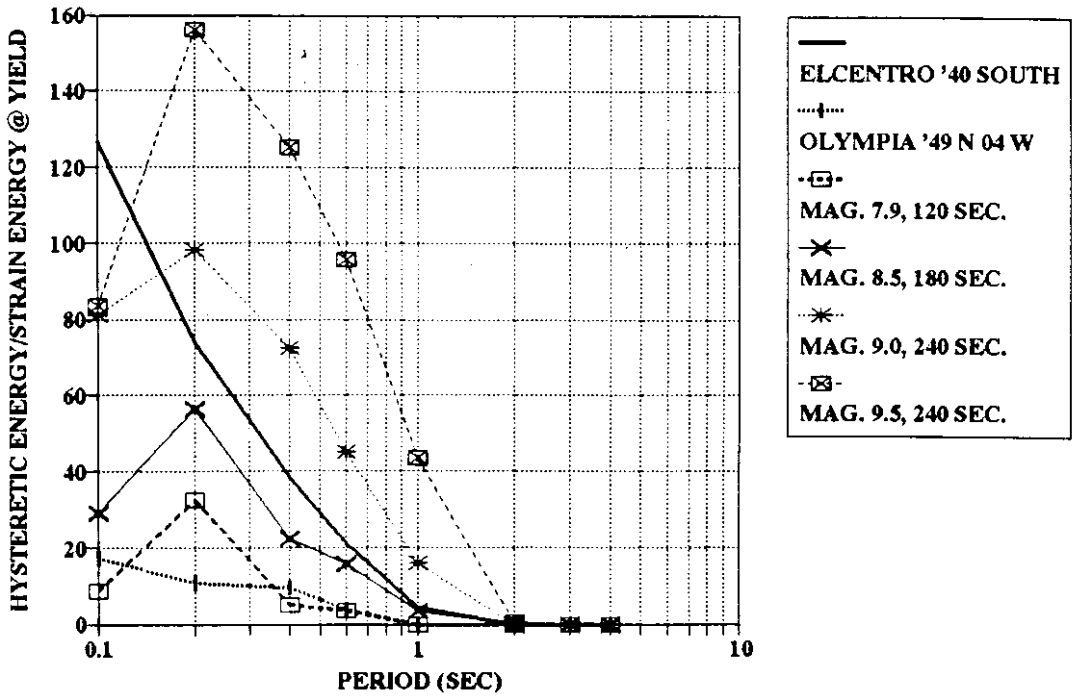


Figure 13 Hysteretic Energy Demand - Puget Sound, Fy/W = 0.25, Degrading Stiffness

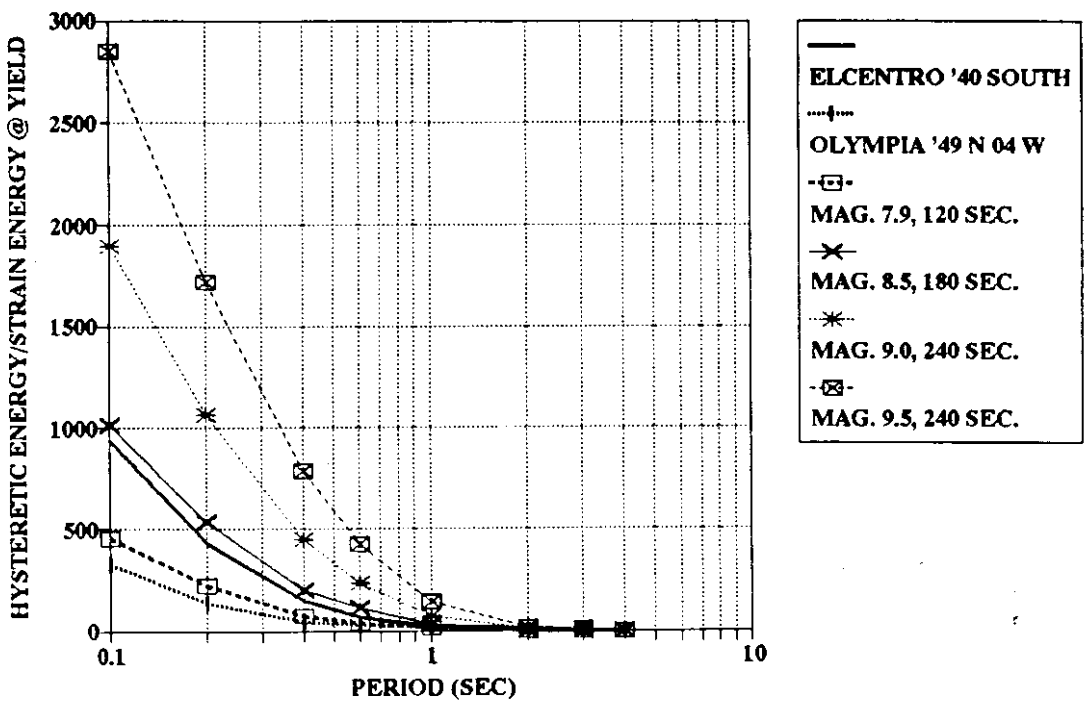


Figure 14 Hysteretic energy Demand - Puget Sound, Fy/W = 0.125, Degrading Stiffness

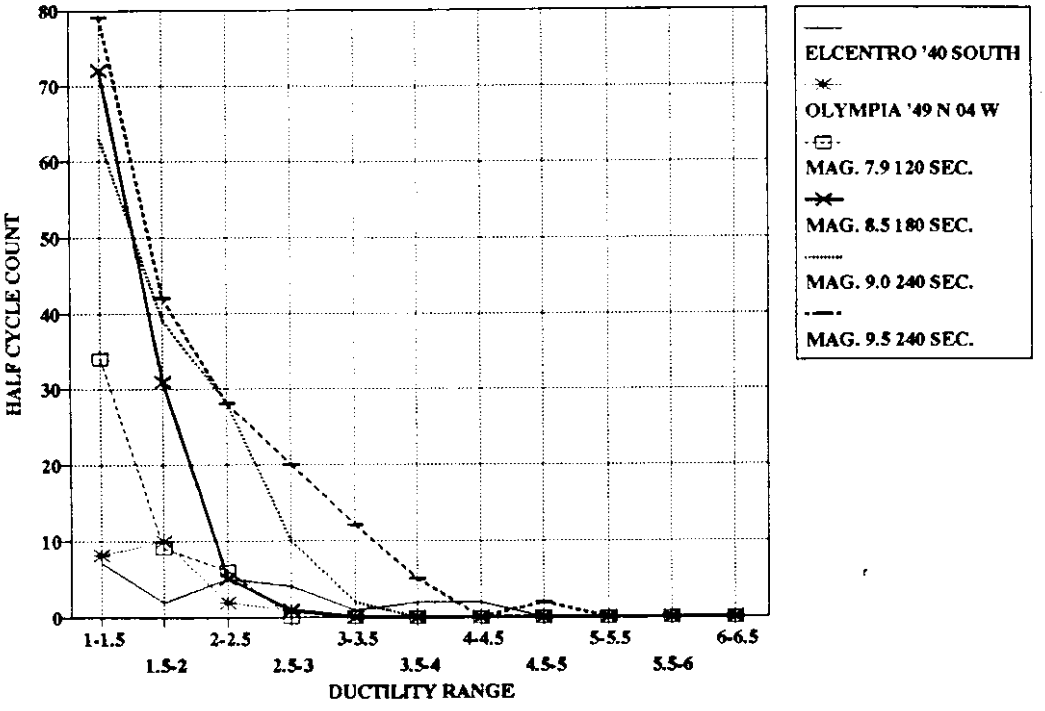


Figure 15 Half-Cycle Counts - Fy/W = 0.25, Degrading Stiffness, Period = 0.6 seconds

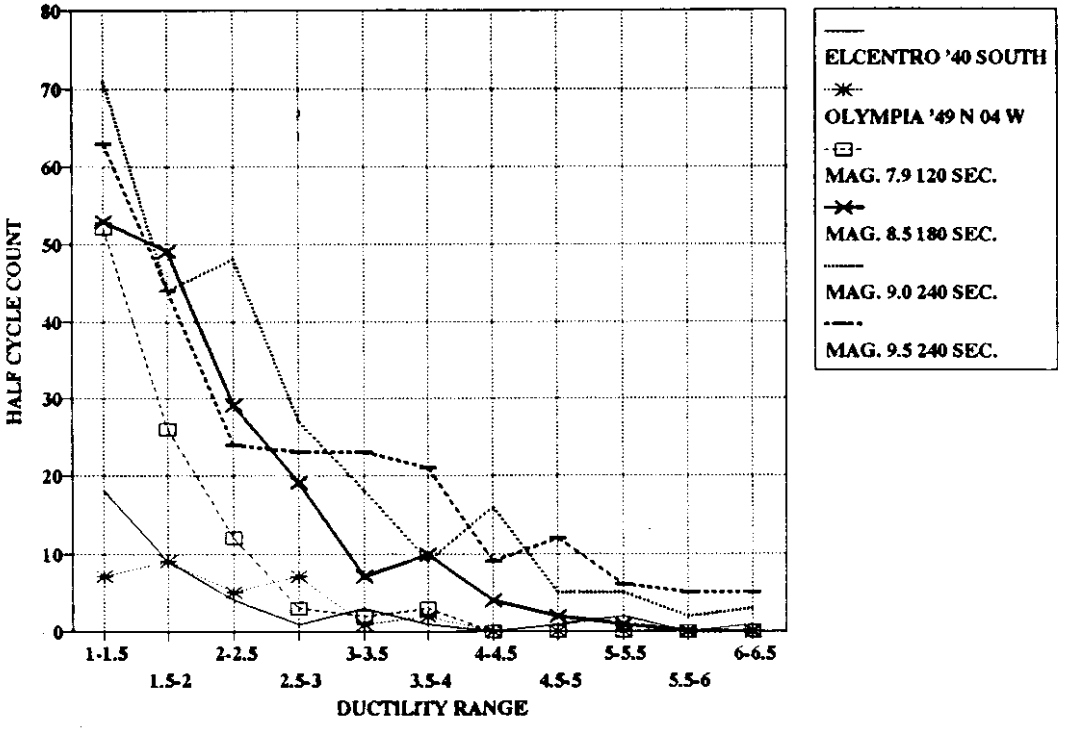


Figure 16 Half-Cycle Counts - Fy/W = 0.125, Degrading Stiffness, Period = 0.6 seconds

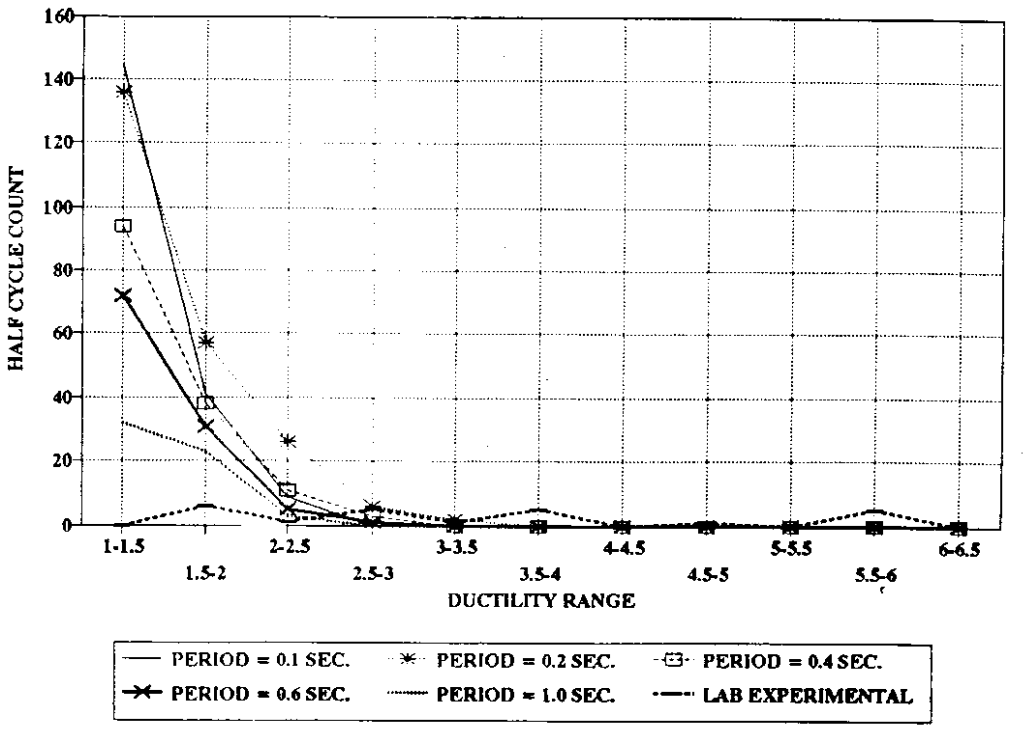


Figure 17 Half-Cycle Counts - Fy/W = 0.25, Degrading Stiffness, Magnitude 8.5

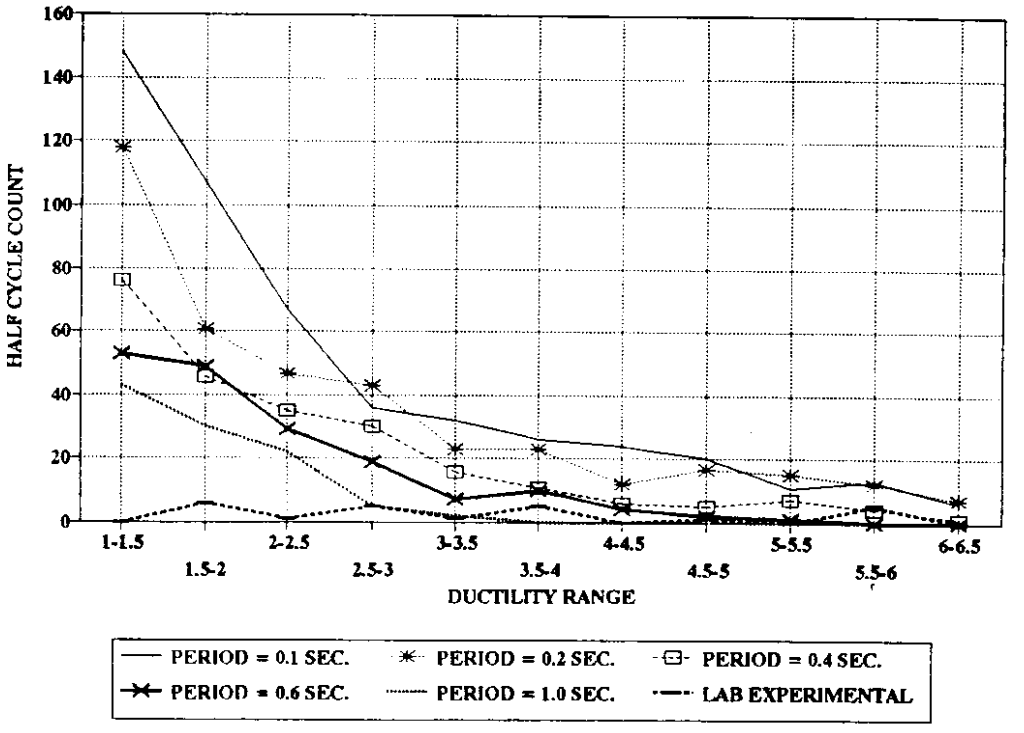


Figure 18 Half-Cycle Counts - Fy/W = 0.125, Degrading Stiffness, Magnitude 8.5

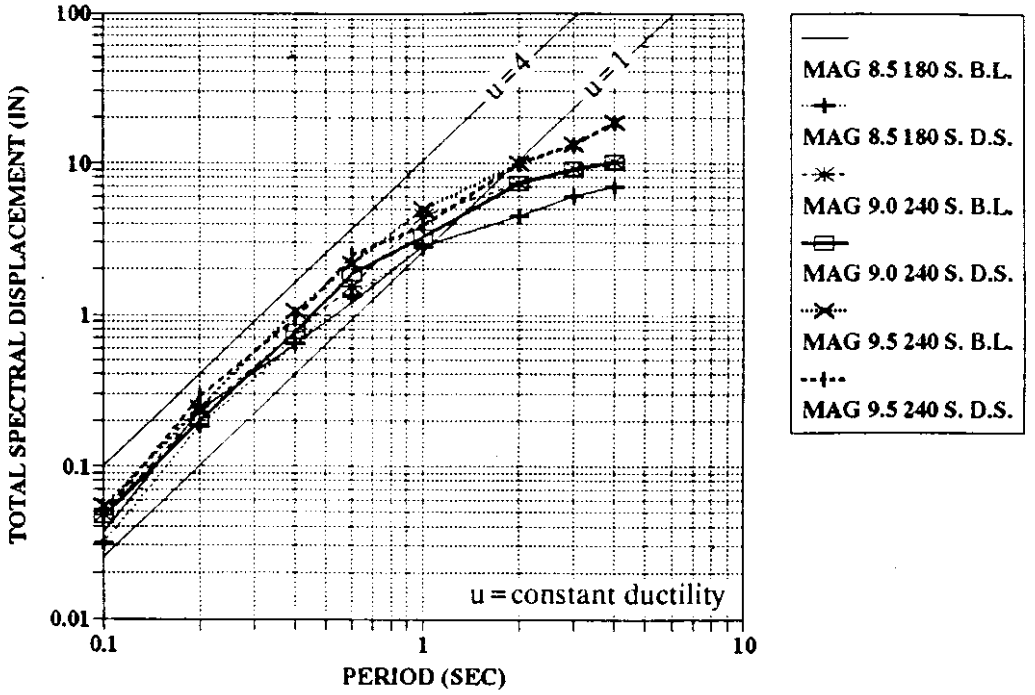


Figure 19 Maximum Displacements - Bilinear vs. Degrading Stiffness, Fy/W = 0.25, Puget Sound

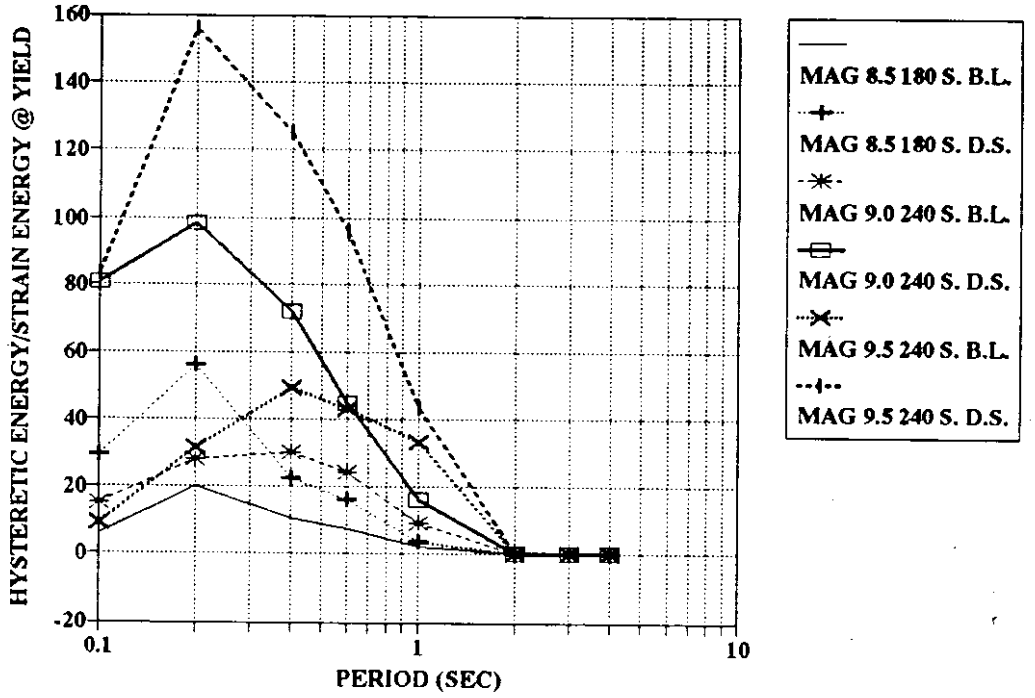


Figure 20 Energy Demands - Bilinear vs. Degrading Stiffness, Fy/W = 0.25, Puget Sound

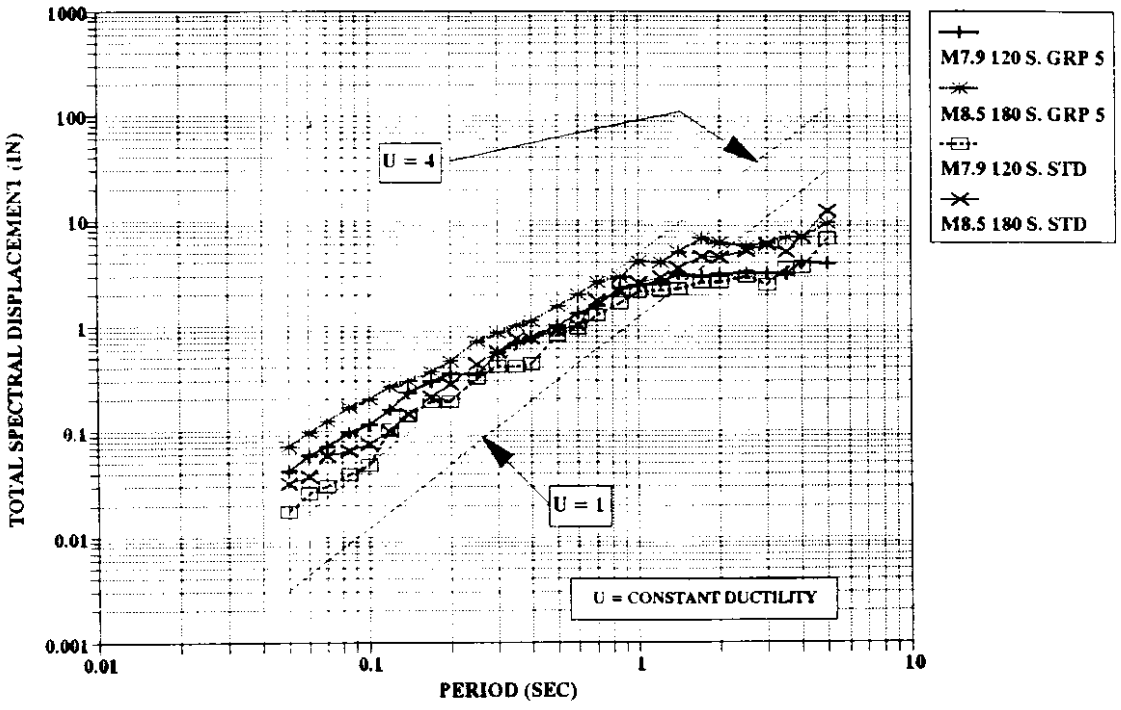


Figure 21 Maximum Displacement - Group 5 Soil, Fy/W = 0.125, Puget Sound

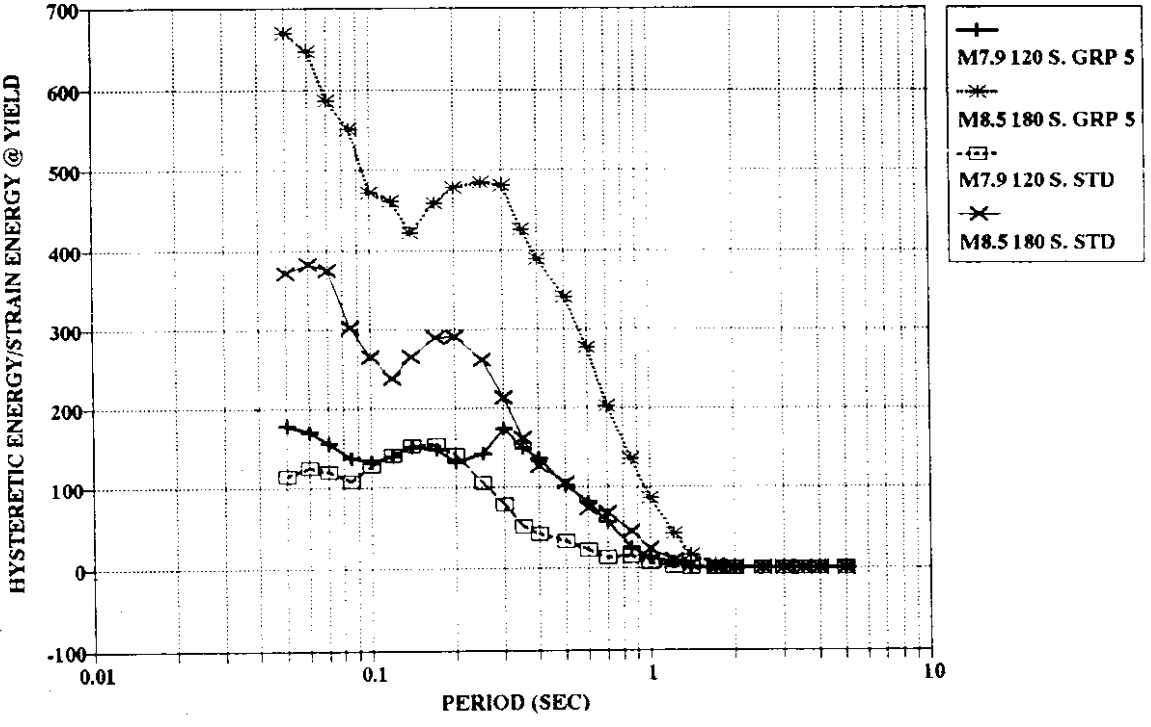


Figure 22 Energy Demand - Group 5 Soil, Fy/W = 0.125, Puget Sound

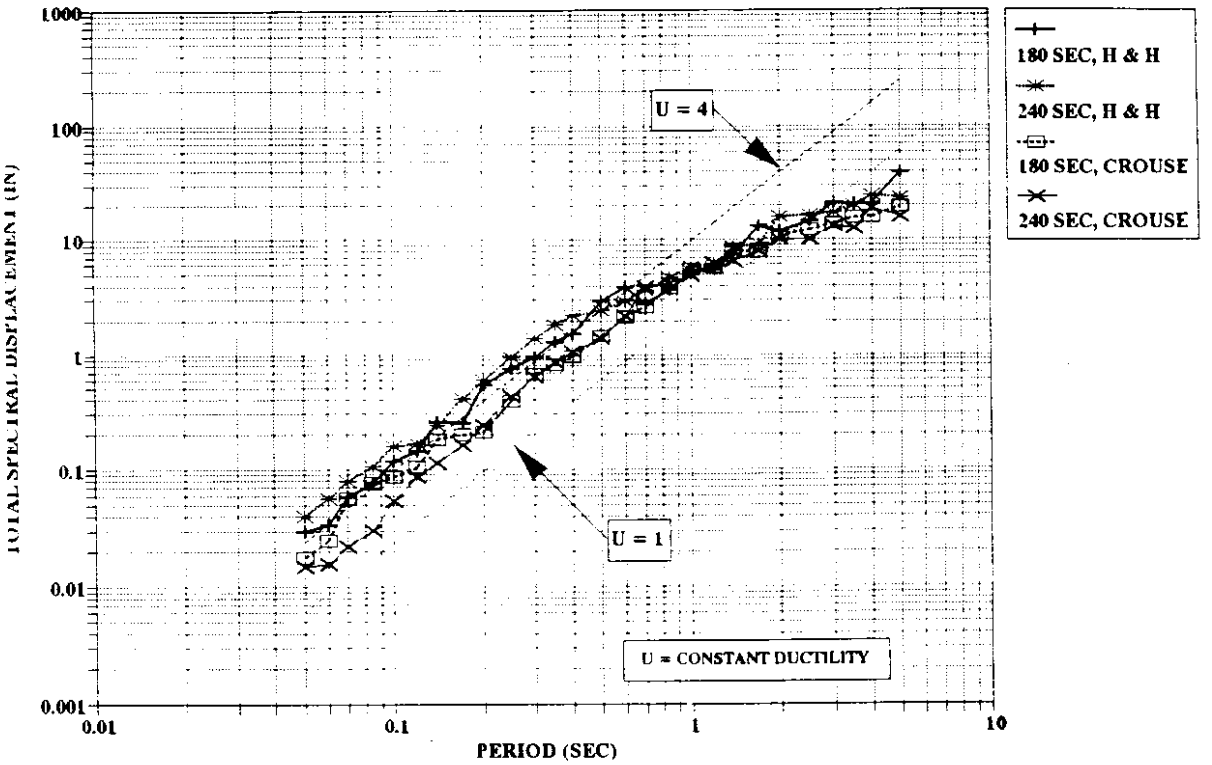


Figure 23 Maximum Displacement - Different Input Spectra, Fy/W = 0.25, Magnitude 9.5, Puget Sound

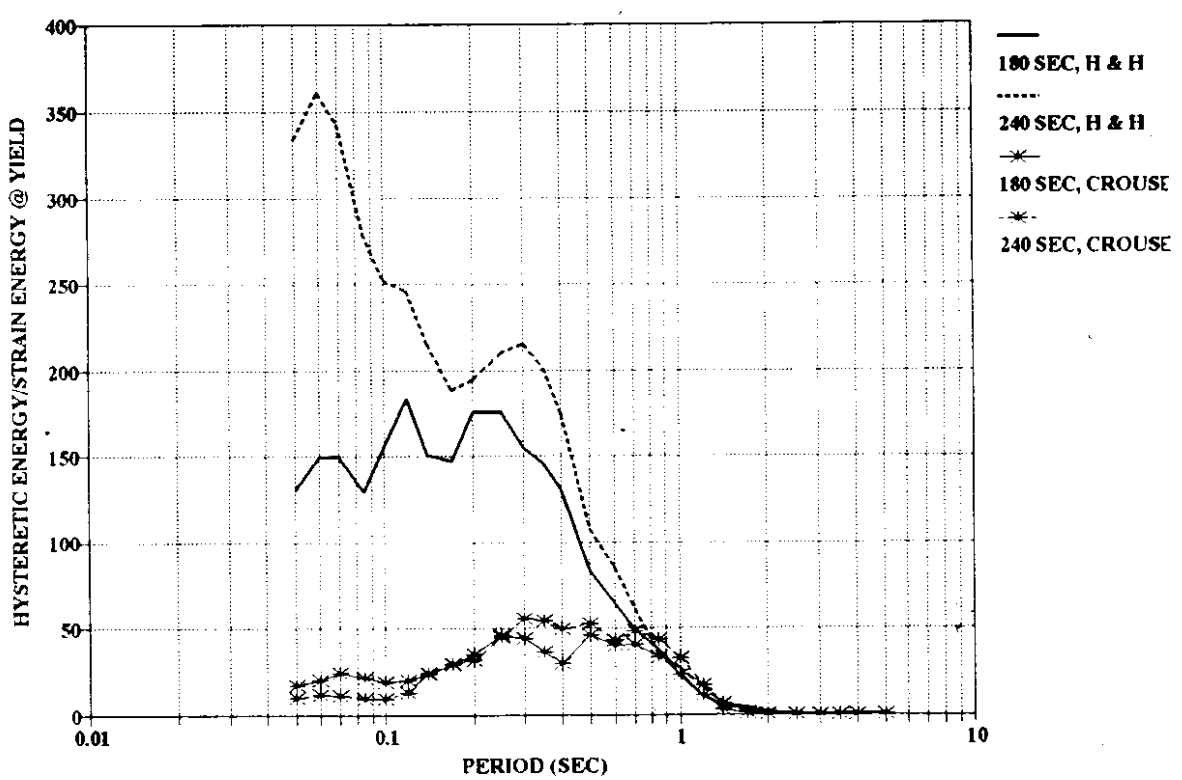


Figure 24 Energy Demands - Different Input Spectra, Fy/W = 0.25, Magnitude 9.5, Puget Sound

CHAPTER 7

APPLICATIONS / IMPLEMENTATIONS

7.1 Applications

Results from this research may be used to estimate the CSZ earthquake demands that may be placed on an existing structure, and the results may be used during design to limit the demands expected on a new structure. Caveats to application of the work at the present time are: the structure must be generalized as a single-degree-of-freedom (SDOF) structure and its force-displacement relationship must be idealized as either bilinear or degrading stiffness.

To estimate the demands on an SDOF structure, the mass that will contribute to inertial loading must be known, as must be the period of vibration and the yield force to weight ratio, Fy/W. The load-displacement relationship can be determined from a "push-over" analysis. Typical results will not be bilinear; thus the actual load-displacement curve must be approximated with an equivalent bilinear curve. Once the necessary input information has been assembled, the plots provided in this report and the accompanying technical report may be used to estimate the inelastic CSZ demands. In general, the curves for bilinear structures should be used for steel structures and those for degrading stiffness used for reinforced concrete structures.

Figure 25 shows an example of determining demands for a structure located in the Puget Sound area and subject to an acceleration record for a 3-minute, magnitude 8.5 event. The period of the SDOF system is 0.6 seconds, the weight is 100 kips, the yield force is 25 kips and the yield displacement is 0.88 in. Using the curve for the given record, the maximum displacement that this structure experiences is 1.5 inches. This corresponds to a ductility demand of 1.7, not a particularly large value.

Using Figure 26 the energy dissipated by hysteretic or yield mechanisms is 1.8 in-kips/kip of structure weight. In this case the total energy is 180 in-kips. The energy stored at first yield is 22 in-kip; thus the structure dissipates a total of 8.2 times this amount over the duration of the acceleration record. The energy dissipated during one complete load reversal sequence at a ductility of 2 can be used to normalize the total energy the structure dissipates over the entire record. If this is done, the energy demand is equivalent to that of 2.1 cycles applied at a ductility of 2. In this case, this is not a particularly large value. A more complete description of this process is contained in the technical report.

The application to the design of a new structure follows essentially the same process in reverse. Here a strategy might be to set the total energy to an equivalent number of cycles at a given ductility demand, then use the plots to determine a Fy/W ratio that will limit the demand to the desired level.

In both applications it is suggested that linear interpolation be used between parameters that are specified in this report. For instance, interpolation may be used for results between Fy/W ratios of 0.125 and 0.25. Based on the uncertainties in the ground motion and without additional, more detailed information, this is a rational approach.

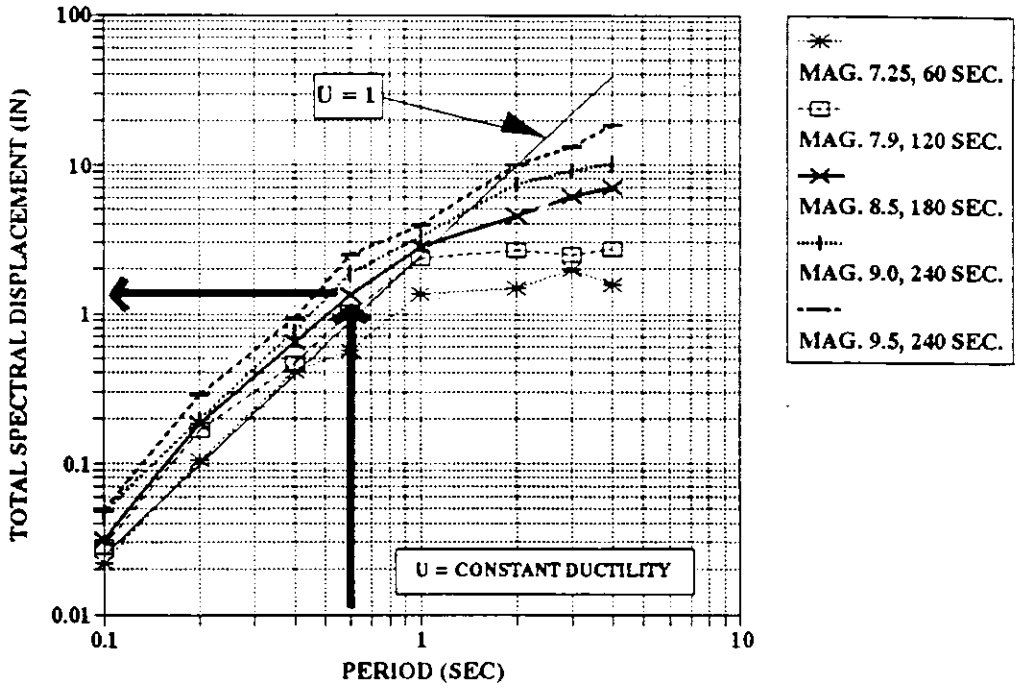
7.2 Implementation

As more becomes known about the Cascadia Subduction Zone and its ability to generate large magnitude earthquakes the information reported here will be useful for updating seismic design code provisions. In light of the current uncertainties regarding the earthquake generation potential of the CSZ and the similarities in the inelastic demands produced by the mid-range artificial CSZ records and recorded events, such as El Centro and Olympia, changes to the current design procedures are not recommended at this time.

Location: Puget Sound Basin Type: Reinforced Concrete

T = 0.6 sec. W = 100 kips F_y = 25 kips Δ_y = 0.88 in.

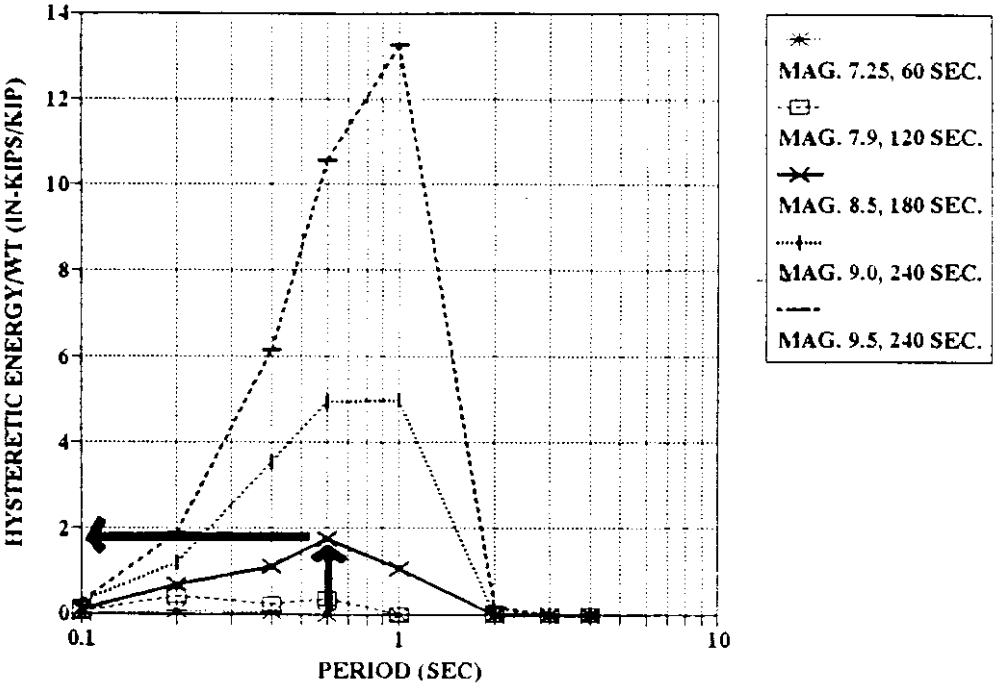
Magnitude 8.5 , 3 Minute , CSZ Earthquake



Maximum Displacement ..... $\Delta_{max.}$ = 1.5 in.

Ductility Demand ..... μ = 1.7 ($\Delta_{max.}/\Delta_y$)

Figure 25 Assessment of Displacement Demands on an Existing Structure



Energy Dissipated / Weight..... E/W = 1.8 in-kips/kip

Total Energy Dissipated..... E_{total} = 180 in-kips
(100 x 1.8)

Energy Stored at Yield ..... E_y = 22 in-kip
(25x0.88/2)

Ratio of $E_{\mu=2}$ /E_y ..... $E_{\mu=2}$ / E_y = 3.9

Equivalent Number of Cycles at a Ductility
Demand of 2 ..... N = 2.1 (180 / 22 x 3.9)

Figure 26 Assessment of Energy Demands on an Existing Structure

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